CONCRETE
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CONSTRUCTIONAL ENGINEERING

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DECEMBER, 1955. **Vol. L, No. 12**

FIFTIETH YEAR OF PUBLICATION

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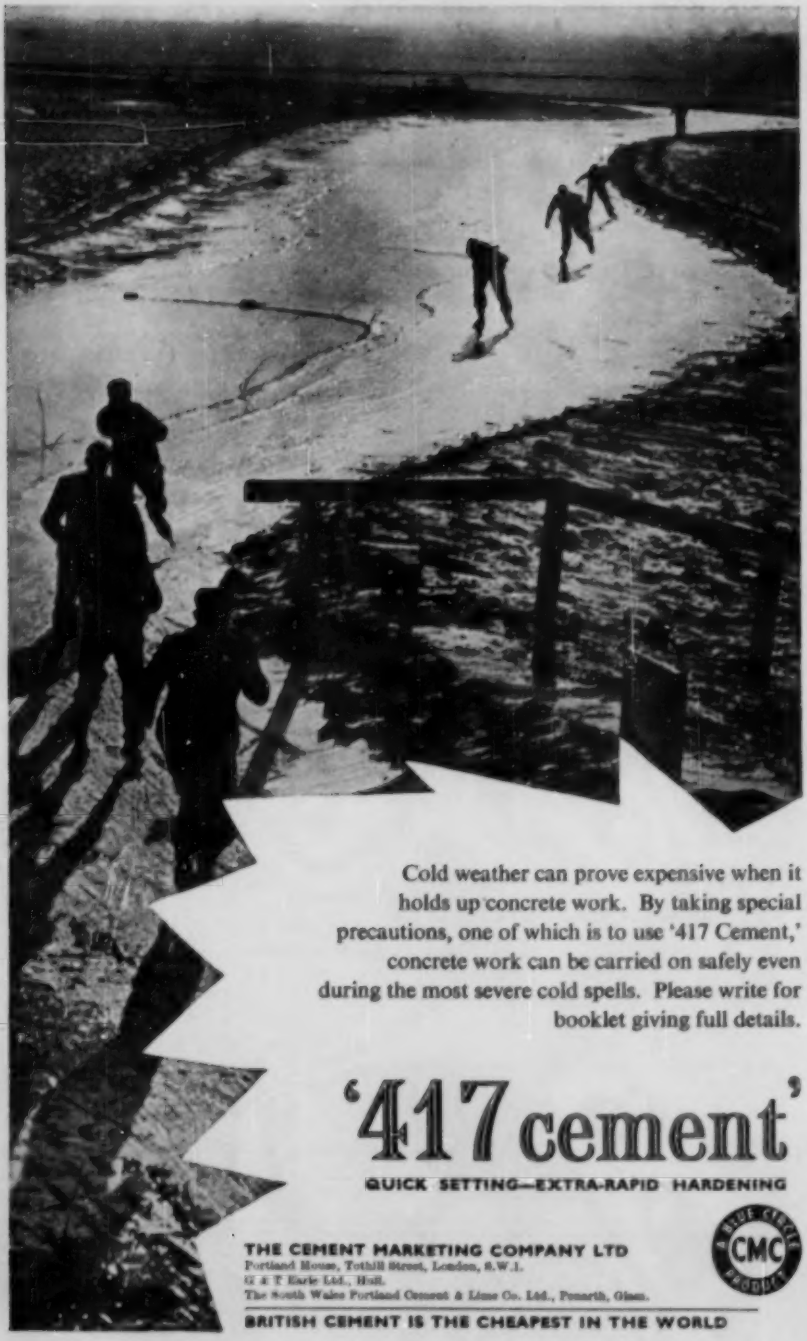

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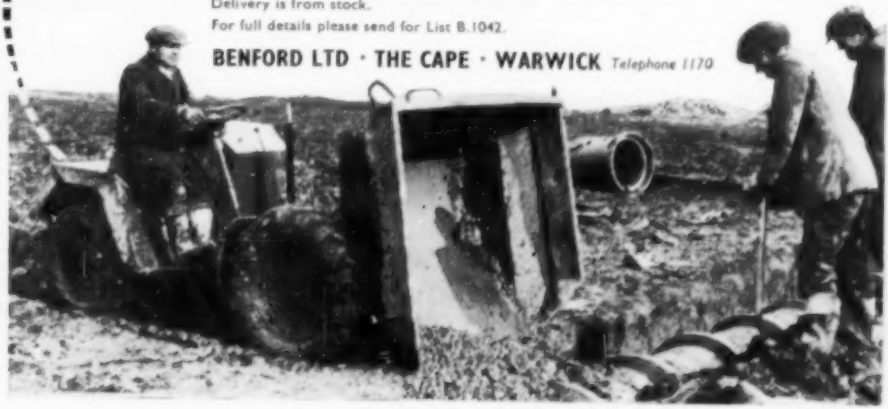

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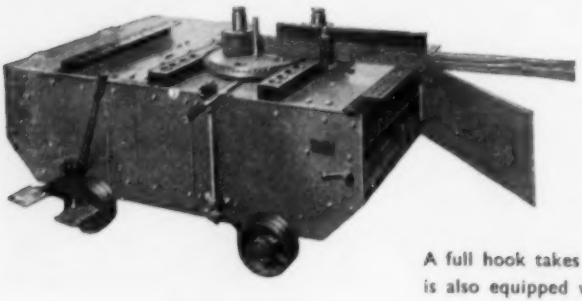

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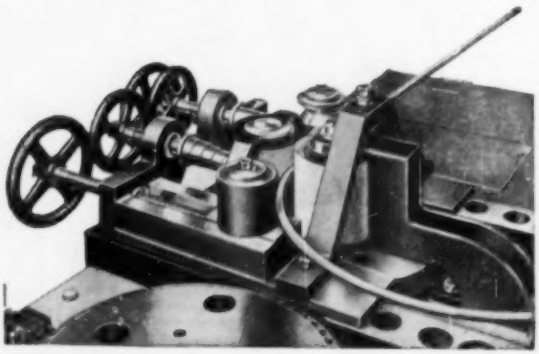

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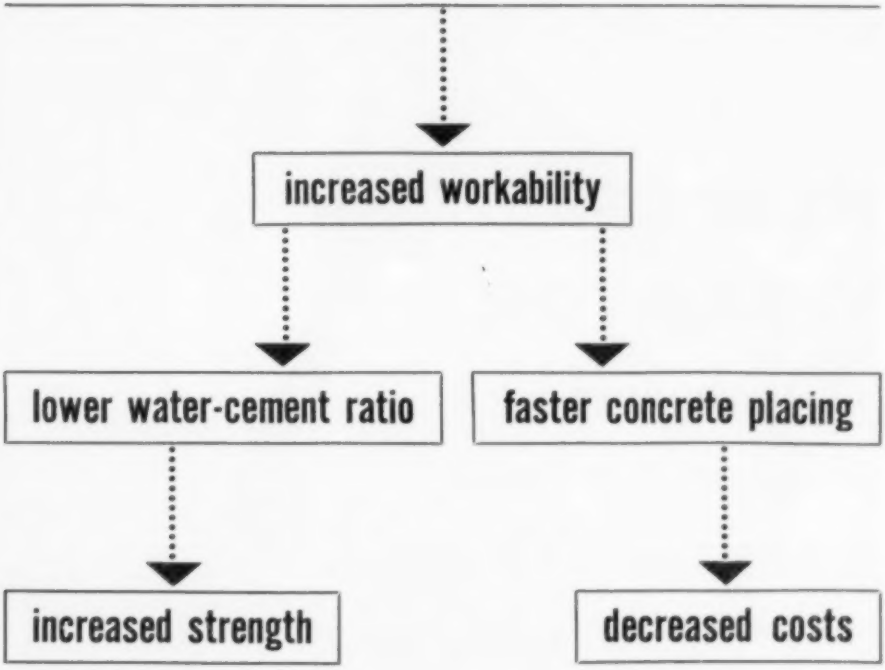

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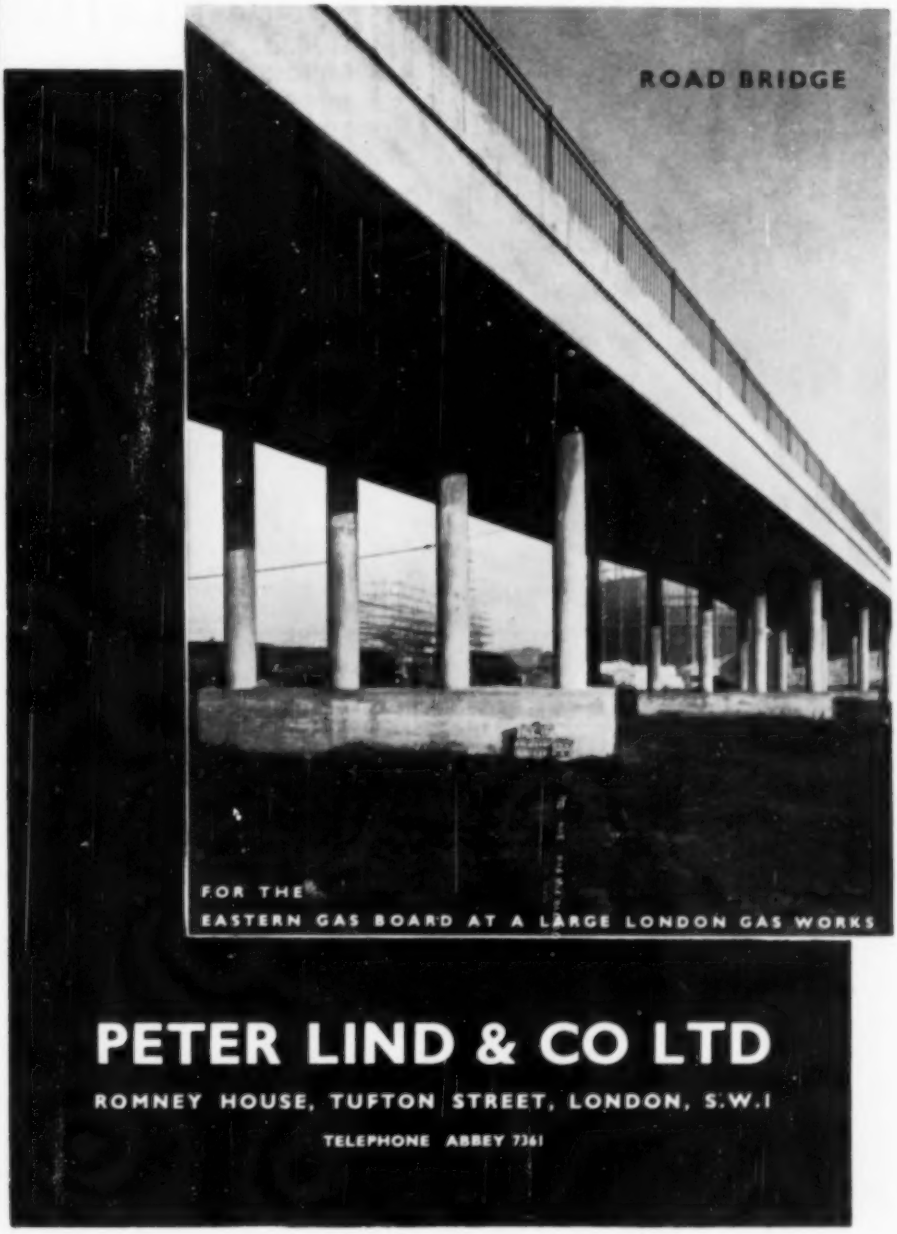
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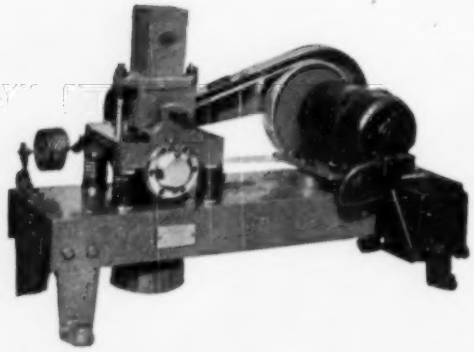

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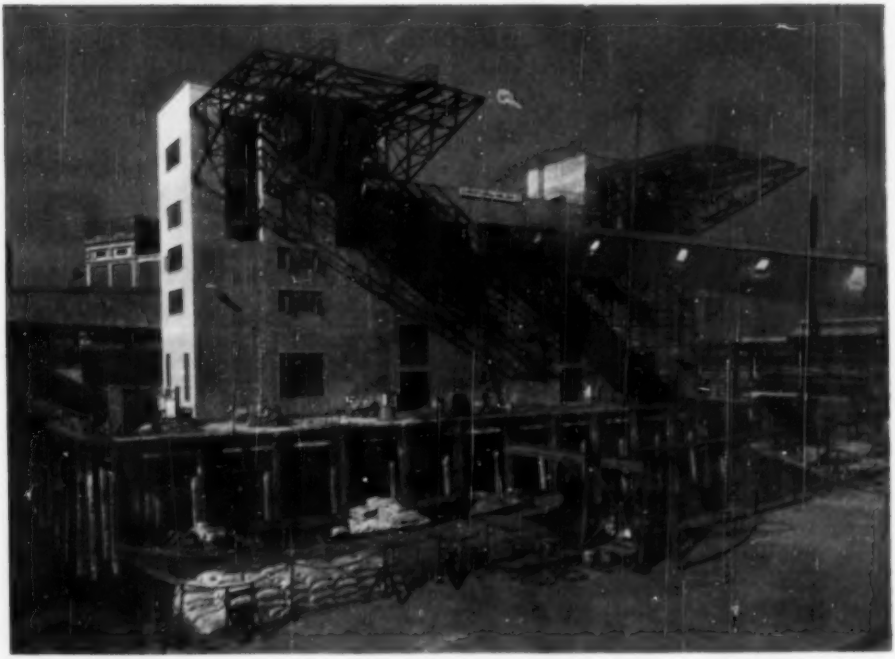

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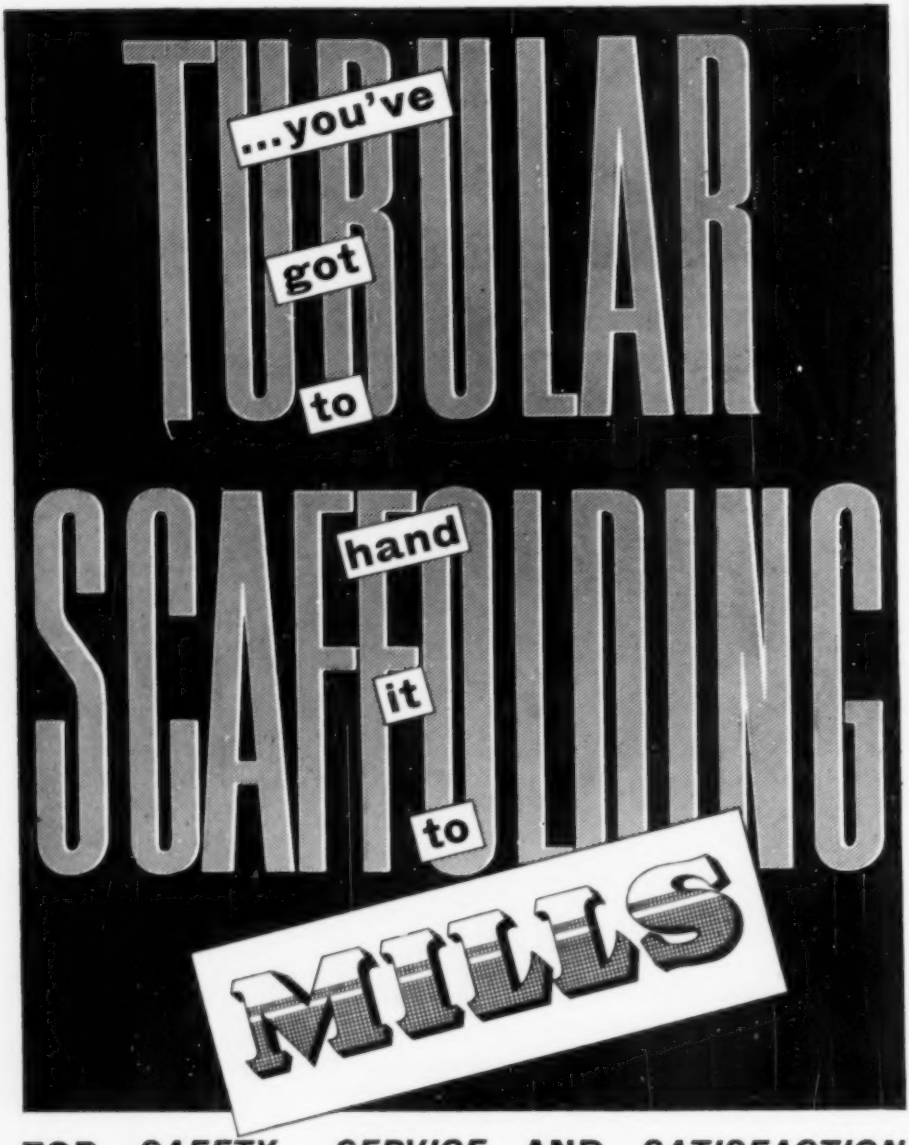

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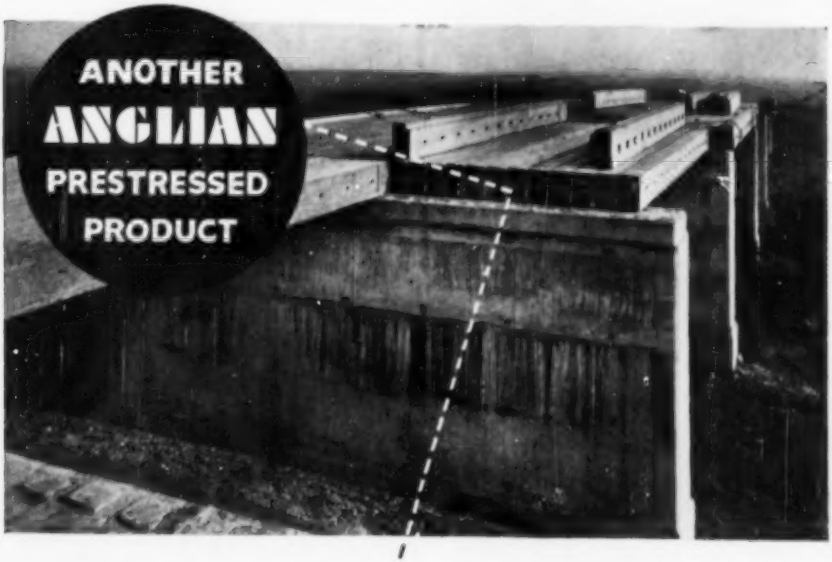

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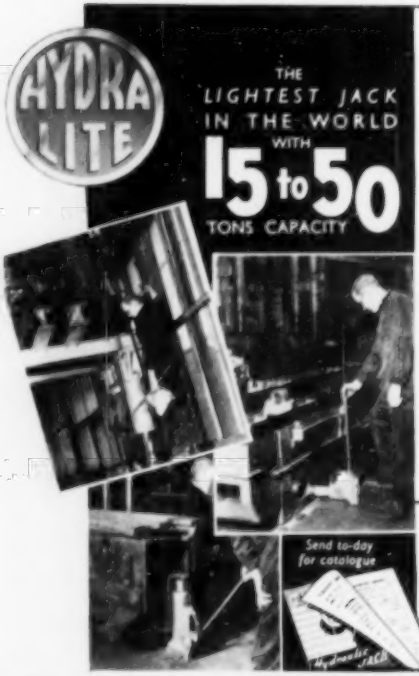

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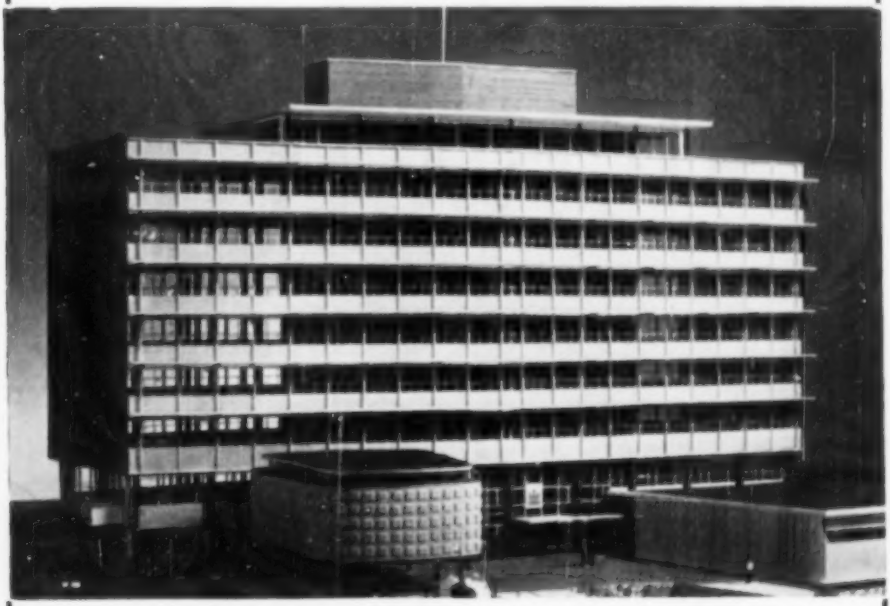

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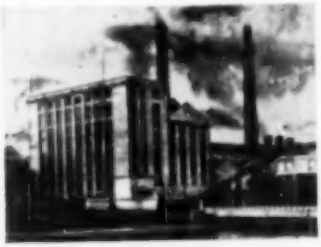

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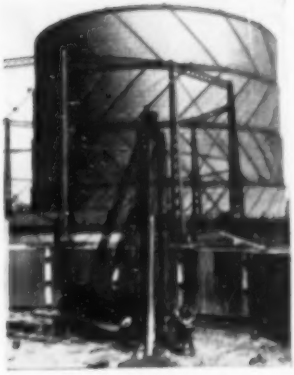

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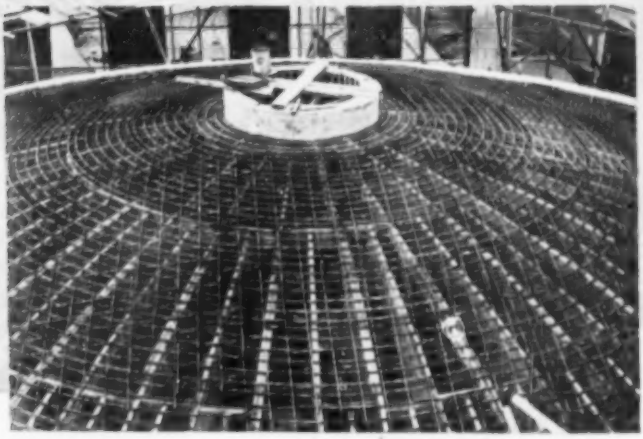

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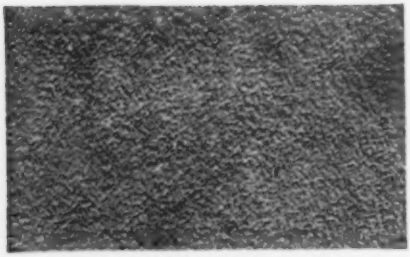
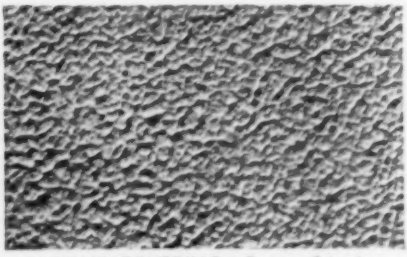


STONE COVERING—Fine Stipple



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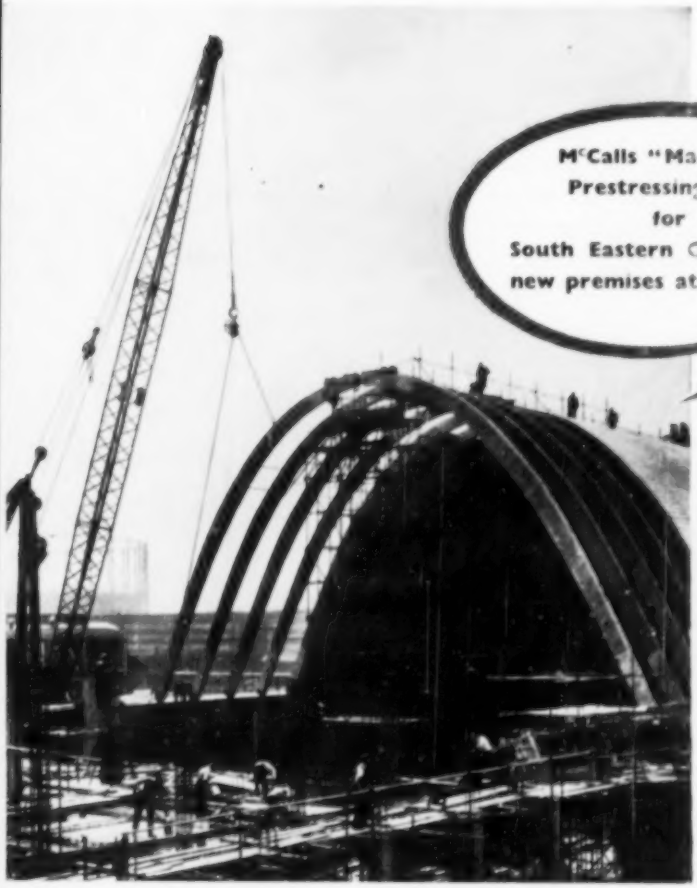

Precast Arch-Ribs — Lee-McCall post-tensioned units

The photograph shows a new sulphate of ammonia store and bagging house for South Eastern Gas Board at Phoenix Wharf, Greenwich.

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Prestressed concrete sections made with Ciment Fondu and used in the construction of a tobacco factory at Bristol.
By courtesy of C. W. Glover & Partners

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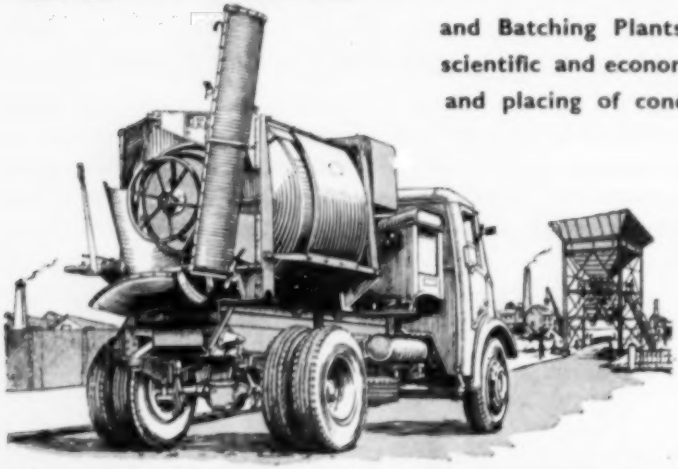

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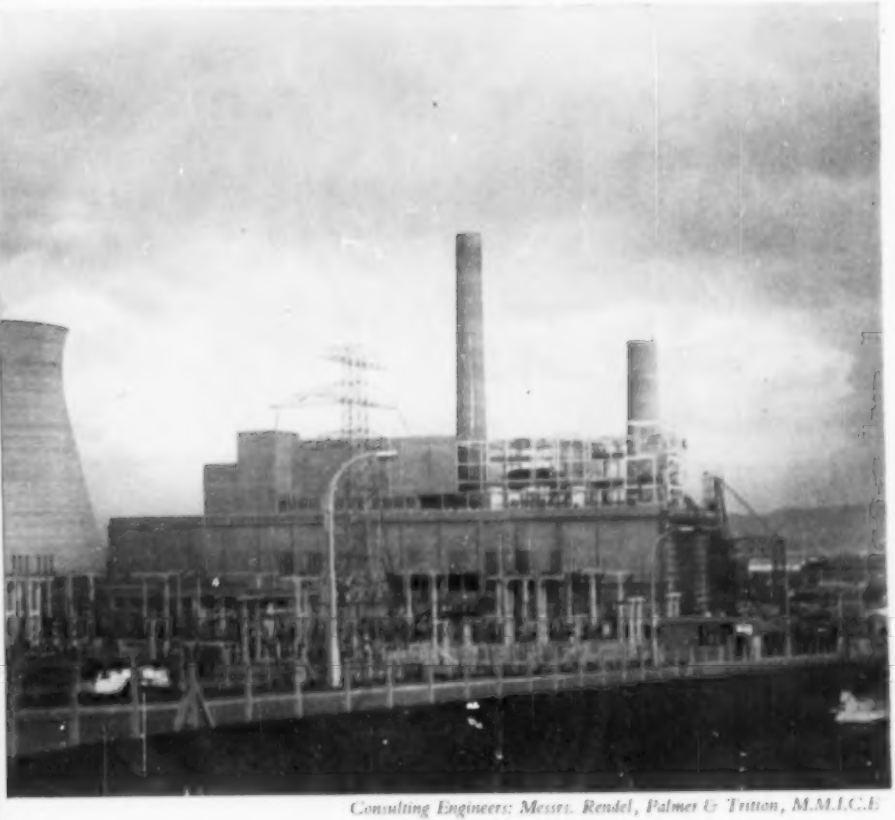

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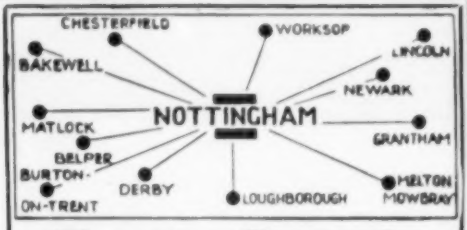

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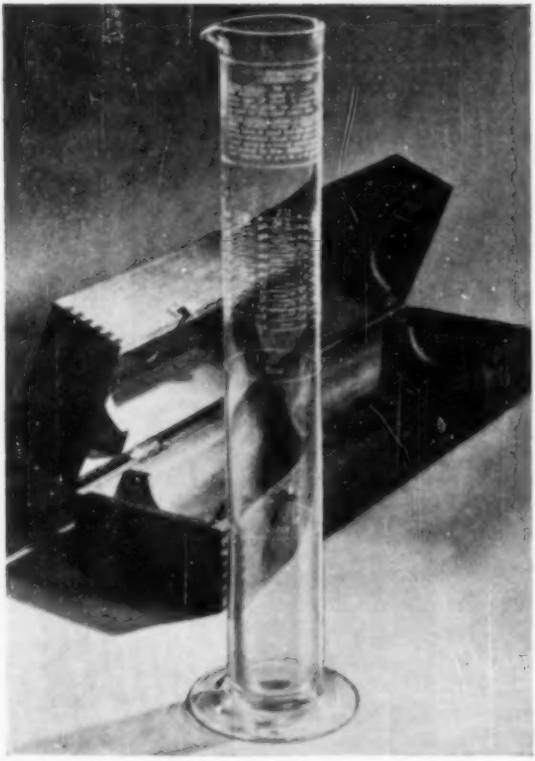

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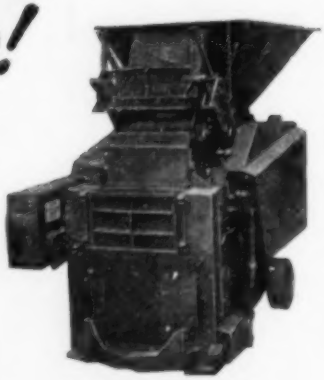

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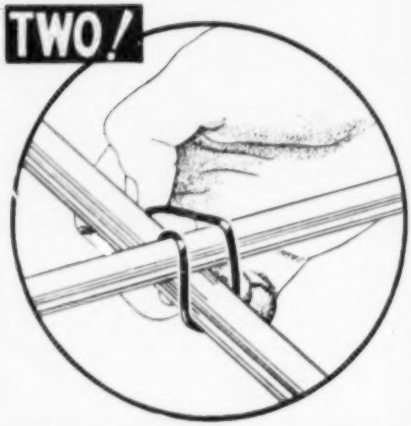

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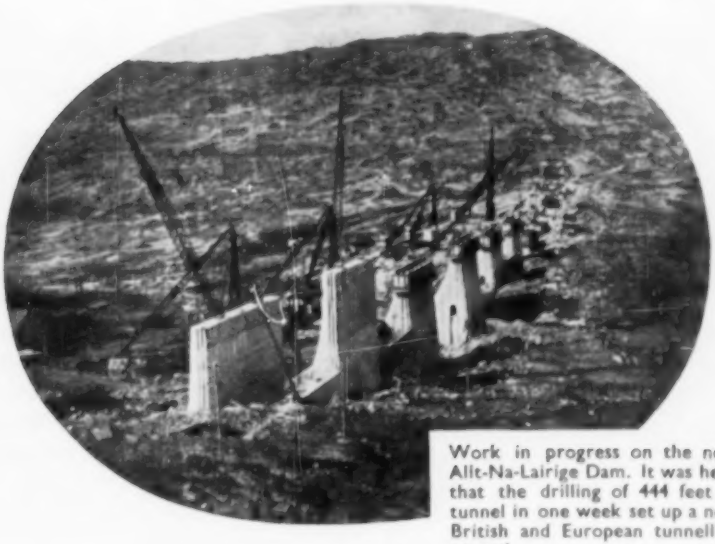

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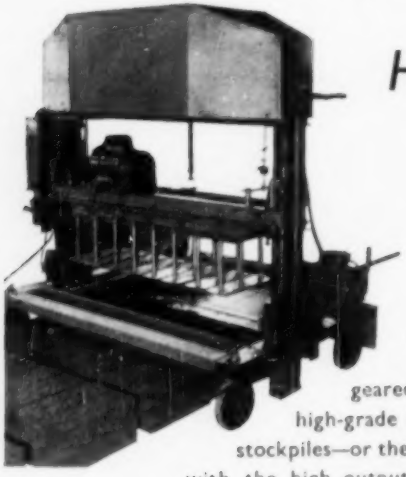

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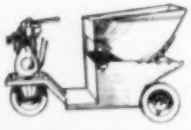

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Volume L, No. 12. LONDON, DECEMBER, 1955.

EDITORIAL NOTES

Kips and Inks.

IN our September number we deplored the introduction into printed matter of contractions which, if they are useful at all, are more suitable for use by an engineer in making his own notes, and we particularly referred to the American contractions psi and kip. That psi is intended to mean pounds per square inch there can be little doubt, but for the sake of readers who are not familiar with it we still prefer to go to a little extra trouble and use the familiar lb. per square inch. In the case of kip, however, there seems to be already much confusion. The note in our September number was written after reading two manuscripts in which kip was used for moments in terms of a thousand inch-pounds and k was used for loads in terms of one thousand pounds. In the latter case the letter k is the initial letter of kilo, and there is nothing to show that it means anything more than one thousand, as do the metric kg. for kilogrammes, km. for kilometre, and so on. In the case of kip it has been assumed that, as in the case of psi, each letter is the initial letter of a word, that is k is for kilo, i for inch, and p for pounds, and that consequently a kip is a thousand inch-pounds. We are assured, however, that this is wrong, and that kip as well as k mean only one thousand pounds. Mr. Aron L. Mirsky, Assistant Professor of Civil Engineering at the University of Connecticut, tells us that so far as he knows and has been able to find out " The term kip is restricted to the meaning 1000 pounds, and the abbreviation is k. By extension from the abbreviation psi for pounds per square inch, kips per square inch is written as ksi. There is no standard abbreviation for thousand inch-pounds. What is commonly used is in.-k or in.-kips ". It appears, therefore, that we have inadvertently added to the confusion by assuming that writers who use kip to mean one thousand inch-pounds were following the accepted American usage. It now appears that one thousand inch-pounds may be variously written ksi, in.-k, or in.-kip, but not kip.

We still do not think that such abbreviations are worth while. How much quicker is it to write 2·5 in.-kip than 2500 in. lb. ? If we count the full points there are ten characters in each of these terms, so there is no saving at all in putting pen to paper or in the quantity of ink used. But in the new style there is much more risk of overlooking the decimal point or forgetting to add the noughts. It seems to us quite ridiculous to use such symbols compounded of the initial letters of French kilos and British pounds where there is no saving in time or in writing to compensate for the resulting confusion and possibility of error.

In our September number we gave some examples of how writers on technical matters waste ink and time by writing unnecessary words, sometimes to such an extent that they have to be read several times before their meaning becomes clear. Long and high-sounding words are often used when shorter ones would be better. For example, it is common for engineers to say and write approximately, or approx., when the shorter word about is meant and should be used; we have actually seen "approximately about". It also seems to be thought that of the order of sounds better than about; this may be so, but there is no excuse for writing four words when one is sufficient and at the same time search for abbreviations that lead to confusion and do not save a single stroke of the pen, as happens when 2·5 in.-kips is used instead of the unmistakable 2500 in.-lb. Approaching is another wasteful word when it is used in the sense of nearly, and so is executed when built is meant. Another unwanted word is introduced in the revised edition of the British Standard for Concrete Pipes recently issued, where the crushing test is described as the crushing proof test—why proof? The use of the words "the value of" is another example of the use of unnecessary words. Why write "the value of" the load is 20 tons, when all we mean is that the load is 20 tons? One of the commonest uses of a blue pencil is to draw it through these three unnecessary words. The frequent use of the words equal to is in the same category. Why write "the bending moment is equal to 1000 ft.-lb." when it is sufficient to write "the bending moment is 1000 ft.-lb."? There is some excuse for using many words to say very little when one is paid for his writings according to the space they occupy on a printed page, but none for a busy engineer to whom time is valuable. How stupid to invent kip as a contraction for one thousand pounds and write "the load is equal to 2·5 kips" instead of "the load is 2500 lb.", or to write "the stress is of the order of 1000 psi" instead of "the stress is about 1000 lb. per sq. in."; in neither case is there any saving in using unfamiliar contractions that may easily lead to errors.

Much misplaced ingenuity has been applied to the invention of labour-saving devices in technical books. At one time it was fashionable to print □″ for square inch and □′ for square foot. More recently we have seen in printed books square inches denoted by #″ and square feet by #′, but it may be that these characters were carelessly drawn squares that the printer interpreted as cross hatching or the preliminaries to games of noughts and crosses.

There has been much discussion in the daily press recently about the inability of many scientists to express their meaning clearly, and the consequent difficulty non-technical people have in understanding the developments in science and technology that so much affect their daily lives, and perhaps their future. Too early concentration on technical training at the expense of education is much to blame for this state of affairs, and it is deplorable that the vast scheme for training technologists at South Kensington deliberately omits the further education of these would-be scientists and technologists—indeed the sponsors of the scheme appear rather to boast that the students' time will not be wasted in achieving an ability to spell their own language and to express themselves in plain words that anyone can understand. However, we see no reason why more strange words that are not to be found in a dictionary should be invented when they effect no saving in time, when well-known terms are already in common use, and when the new terms can so easily lead to error.

The Design of Singly-Reinforced Beams.

By P. WEFRING, B.Sc.

In the May, 1954, number of this Journal was given an interesting description, by Mr. J. S. Savona, of a graphical method of designing beams by means of a circular diagram. The writer does not believe that any graphical method is likely to supersede the existing methods, but an examination of the circular-diagram method may lead to some improvement in present-day practice, particularly in the case of stress analysis, as may be evidenced by the following comparison of methods and the conclusion to which it leads.

The *R* Method.

The most favoured of the methods now in use for the design of singly-reinforced beams and slabs is probably that based on the use of a coefficient R, which has the dimension of stress per unit area, is closely related to the permissible stress in the concrete, is usually to be found in tables, and is defined as $R = \frac{M}{bd^2}$. The use of the R method may be described as follows.

Design.—Given M, c, and t, from a table find the value of R for the maximum permissible stresses c and t for a given mixture of concrete. Calculate $d = \sqrt{\frac{M}{Rb}}$ using a value for b to meet the requirements of a beam of common proportions or of any special requirements. From the same table find p for the maximum permissible stresses c and t. Calculate $A_t = pbd$.

Stress Analysis.—Given M, b, d, and A_t, calculate $R = \frac{M}{bd^2}$, $p = \frac{A_t}{bd}$, and $mp = m.\frac{A_t}{bd}$. The table gives only the maximum permissible stress t in the reinforcement, and is consequently useless for stress analysis. The next step is therefore to find $n_1 = -mp \pm \sqrt{m^2p^2 + 2mp}$, a graph for which is given in many handbooks.

Calculate $a_1 = 1 - \frac{n_1}{3}$. Calculate $c = \frac{R}{\frac{1}{2}a_1n_1}$ and $t = \frac{R}{p.a_1}$.

Circular Design Method.

This method makes use of a dimensionless coefficient α, which is related to R as shown in the following. Mr. Savona starts from $\frac{M}{bcd^2} = \frac{a_1n_1}{2}$ and $\alpha = \frac{2}{a_1n_1}$. Combining these expressions we have $\frac{M}{bcd^2} = \frac{1}{\alpha}$, or $\frac{M}{bd^2} = \frac{c}{\alpha} = R$. Hence $\alpha = \frac{c}{R}$. Thus α provides the relationship previously referred to between c and R.

The use of the circular diagram method is described by Mr. Savona as follows.

Design.—Calculate $r = \frac{t}{c}$ and $n_1 = \frac{m}{m + r}$. Find α from n_1, using a graph

such as Fig. 3 in Mr. Savona's article with a curve illustrating graphically the expression for α in terms of n_1. Calculate $\frac{M}{bc}$.

Draw a circular diagram, beginning with a circle of diameter $\left(\alpha + \frac{M}{bc}\right)$ and finishing with a segmental chord at distances α and $\frac{M}{bc}$ from opposite points on the circle. Half the length of the chord will then be equal to d by the relationship $\frac{M/bc}{d} = \frac{d}{\alpha}$. Find mp from α, using a graph such as Fig. 4 in Mr. Savona's article with a curve illustrating graphically the expression for α in terms of mp. Calculate $p = \frac{mp}{m}$ and $A_t = pbd$.

Note.—The circular diagram is of considerable academic interest, but it is not understood what is the advantage of drawing a diagram for every beam to be designed, in preference to calculating d from $d = \sqrt{\frac{M}{bc}\alpha}$.

STRESS ANALYSIS.—Calculate $p = \frac{A_t}{bd}$ and $mp = m.\frac{A_t}{bd}$. Find α from mp, using the graph in Fig. 4 in Mr. Savona's article. Draw a circular diagram beginning with the segmental chord of length $2d$ and α as part of a diameter at right angles through the midpoint of the chord, finishing the diagram with the construction of a circle through the three points thus obtained. The diameter of the circle less its component part α is then equal to $\frac{M}{bc}$ by the same relationship as before, namely $\frac{M/bc}{d} = \frac{d}{\alpha}$. Calculate c from $\frac{M}{bc}$. Find n_1 from α using Mr. Savona's Fig. 3. Calculate $t = \frac{1 - n_1}{n_1}.\ mc$.

Note.—It is more troublesome to draw the circular diagram for stress analysis than for design, as it has to be drawn in reverse, beginning with chord $2d$ and part diameter α to establish three points on the circle and from these its centre, finishing the diagram with the circle.

The α Method.

As it serves the purpose of design and stress analysis equally well, the writer favours the use of a dimensionless co-efficient α instead of R, but not the use of a circular diagram. Figs. 3 and 4 in Mr. Savona's article can with advantage be combined into one graph with two separate curves for α, as the scales are numerically identical for both curves. One curve will give values for n_1 on the horizontal scale, and the other values for mp. The arrangement of these curves into a single graph makes it possible, in the case of stress analysis, to find n_1 from mp without reference from one graph to another. A specimen design graph is shown in *Fig.* 1.

The conclusions drawn lead to a method of design and stress analysis which may be called " The α Method ".

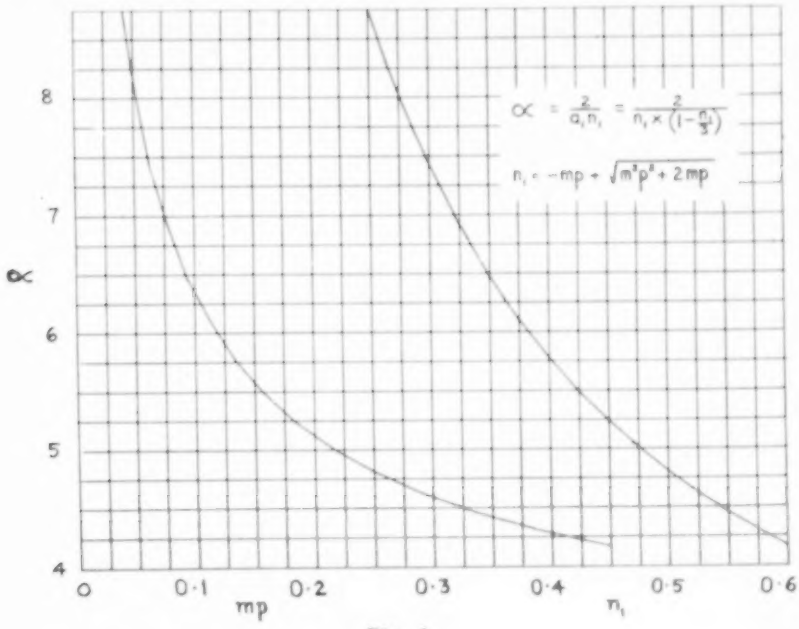


Fig. 1.

DESIGN.—Calculate $r = \frac{t}{c}$ and $n_1 = \frac{m}{m + r}$. Find α from n_1, using the appropriate curve in the graph. Calculate $d = \sqrt{\frac{M}{bc} \times \alpha}$, $p = \frac{n_1}{2r}$, and $A_t = pbd$.

Instead of calculating $p = \frac{n_1}{2r}$, the graph can be used alternatively to find mp at the same time as α by reference to the second curve. Then $p = \frac{mp}{m}$.

STRESS ANALYSIS.—Calculate $p = \frac{A_t}{bd}$ and $mp = m \cdot \frac{A_t}{bd}$. Find α from mp, using the appropriate curve, and at the same time n_1 from α by reference to the second curve. Calculate $c = \frac{M}{bd^2}\alpha$, $r = \frac{n_1}{2p}$, and $t = r \times c$.

Thus a single graph with two curves is all that is needed for rapid design and stress analysis involving only a minimum of simple calculation, and the writer believes the use of the α method in place of the R method and its comparatively elaborate design tables is an improvement on present-day practice.

The Design of Corner Columns.

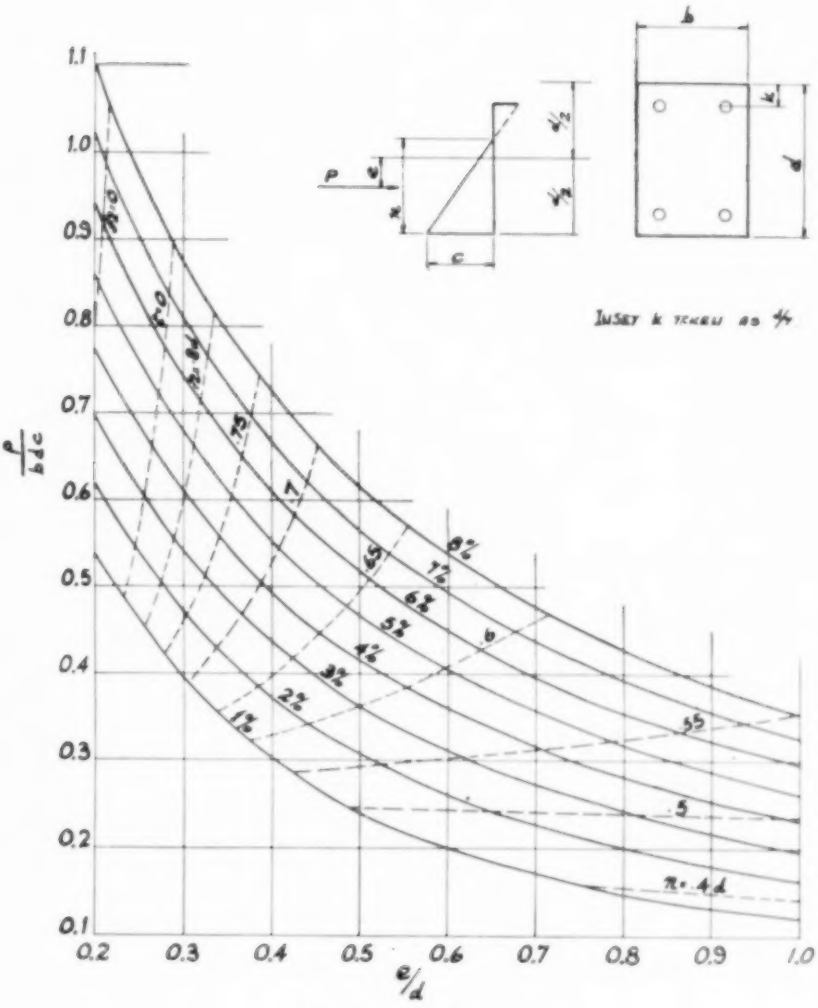
By DOUGLAS W. ALLAN, A.M.I.C.E., A.M.I.Struct.E.

THE method described of designing corner columns subjected to axial loads with eccentricities in two directions at right angles is shorter, although more approximate, than the methods generally used. Most curves used in the design of columns give the percentage of steel required in columns subjected to bending in one direction, with the eccentricity ratio $\frac{e}{d}$ at the base and values of $\frac{p}{bdc}$ as ordinates.

The method here proposed for the design of columns subjected to bending in two directions is based upon the fact that there is a very small difference only between the percentage of steel required in a column subjected to bending in one direction and the percentage of steel required in a column subjected to bending in two directions having the same values of $\frac{p}{bdc}$ and $\frac{e}{d}$, provided that $\frac{e}{d}$ in the two-way case is taken as the arithmetical (not the vectorial) sum of the ratios $\frac{e}{d}$ in the two directions. Within the normal ranges the difference varies between not more than $2\frac{1}{2}$ per cent. below and 5 per cent. above the correct value.

The method proposed consists of the following steps. (1) Compute $\frac{p}{bdc}$; (2) compute $\frac{e_1}{d_1}$ in one direction; (3) compute $\frac{e_2}{d_2}$ in the other direction; (4) add (2) and (3) and increase by $2\frac{1}{2}$ per cent., giving a final value of $\frac{e}{d}$ which is to be read, in conjunction with the value of $\frac{p}{bdc}$, on the curve for bending in one direction, giving at once the safe values required. The curves are given on the facing page; it is assumed that k is $\frac{d}{7}$.

These operations can be completed in one or two minutes, and although in a few cases there will be slightly more steel than is theoretically necessary, the excess is on average not more than about 5 per cent.



Design of Columns.

(See facing page.)

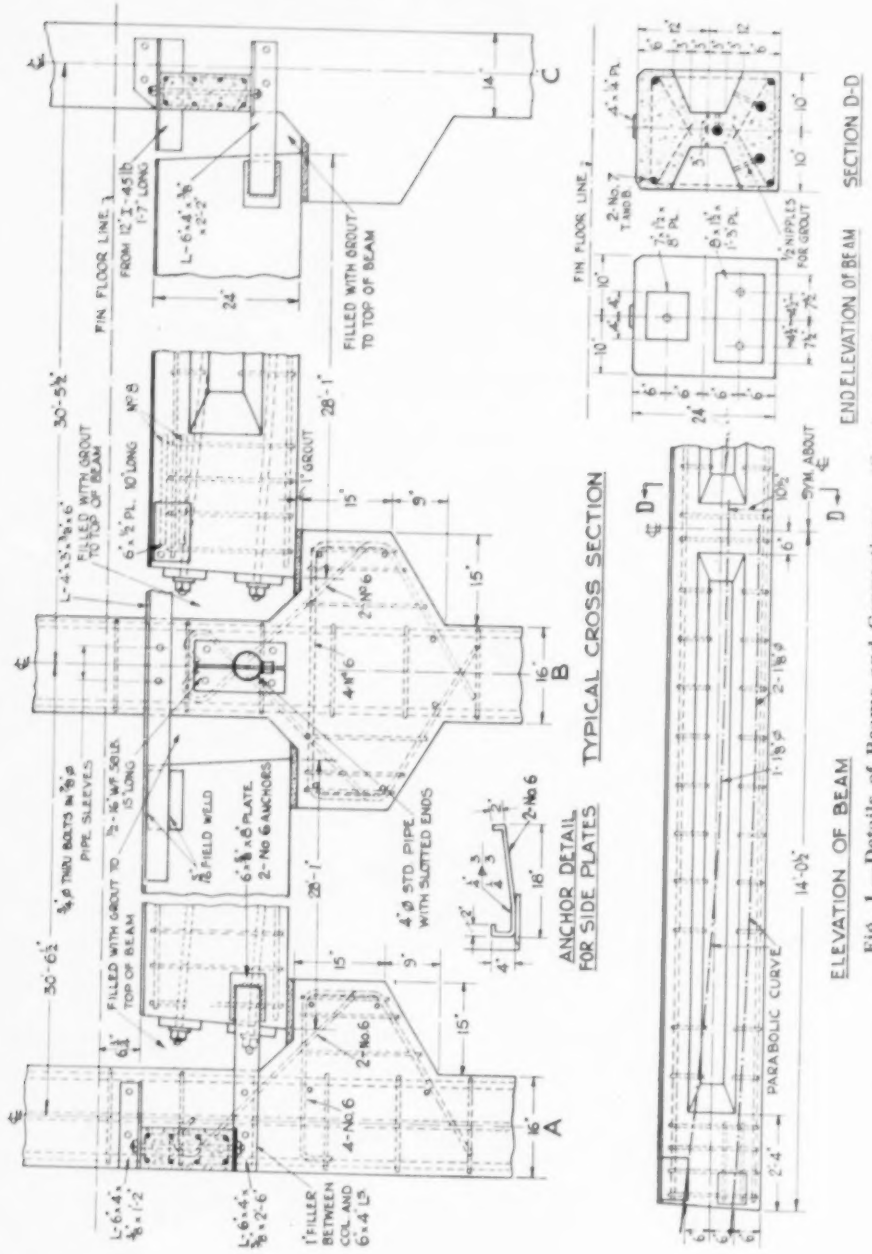


Fig. 1.—Details of Beams and Connections. (See page 421.)

A Prestressed Office Building in the U.S.A.

A CROSS SECTION is shown in *Fig.* 3 of a three-story building recently erected for the Glen L. Martin Co. in Baltimore, U.S.A., in which the columns were precast for the full height of the three stories and the floor-beams spanning 30 ft. 6 in. were precast and prestressed. The roof comprises steel trusses spanning 61 ft. and carrying corrugated steel sheets with insulation boards and bituminous felt. The following description is from " Engineering News-Record ".

The building is 364 ft. long by 62 ft. wide and, as shown in *Fig.* 3, abuts existing buildings on both its longer sides. The precast columns at 16 ft. centres carry floor-beams spanning the width of the building. The external columns are braced in the longitudinal direction by precast concrete struts and the central columns by 4-in. diameter steel pipes welded to plates cast in the columns.

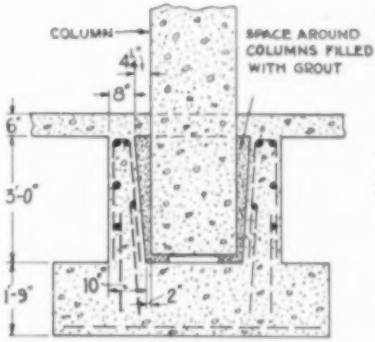


Fig. 2.—Typical Footing.

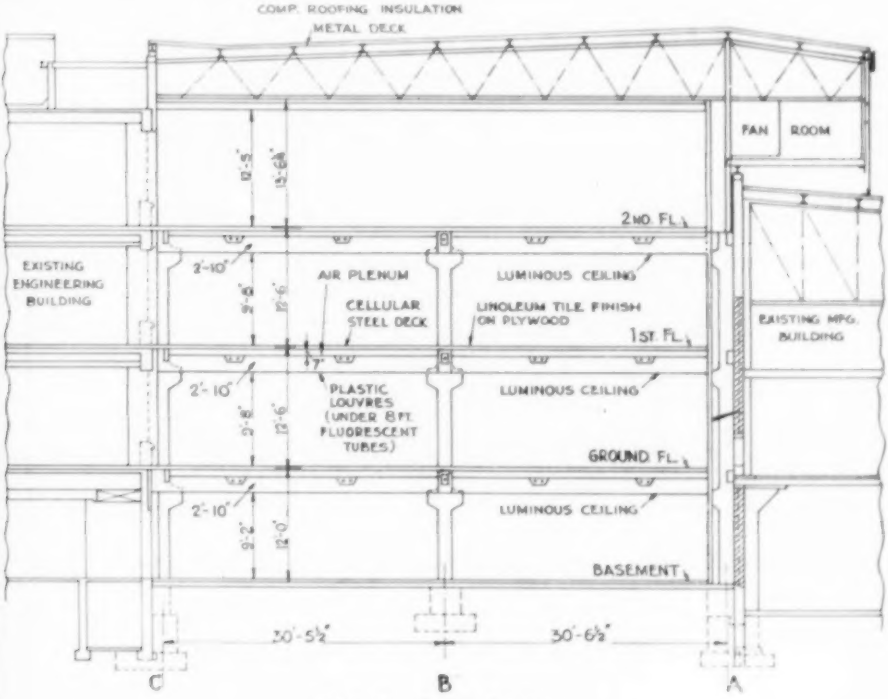


Fig. 3.—Cross Section.

Fig. 4.—Erecting a Column.

The floors are steel cellular members and span between the beams.

The precast reinforced concrete columns are each 40 ft. long and weigh about 12 tons. The columns along one side of the building are 1 ft. 8 in. by 1 ft. 2 in.; on the other side they are 1 ft. 8 in. by 1 ft. 4 in.; and in the centre of the building 2 ft. by 1 ft. 4 in. The brackets supporting the beams project 1 ft. 3 in. Most of the foundations are as shown in *Fig.* 2, but a few columns had steel base-plates welded to the ends of the main bars. One of the columns is shown being erected in *Fig.* 4.

Details of the beams are shown in *Fig.* 1. Generally they are prestressed by three 1¼-in. high-tensile bars anchored by nuts, on the threaded ends of the bars, bearing against steel plates cast in the ends of the beams. The beams are connected to the columns by short pieces of steel angles welded to plates cast in the beams and bolted to the columns (*Fig.* 1, page 420).

All the members were cast adjacent to the site in timber moulds. The members were transported on trailers generally used for transporting parts of aircraft, and were erected by a 30-tons crane.

Ready-mixed concrete was used containing 564 lb. of ordinary Portland cement and 36 gall. of water per cubic yard. The mixture was 1 : 2·1 : 2·6 with the addition of a plasticizing agent, and the largest size of aggregate was ½ in. The crushing strength of 12-in. by 6-in. cylinders at an age of seven days was 4000 lb. per square inch.

The building was designed by the Glen L. Martin Co. in collaboration with Albert Kahn Associated Architects & Engineers, Inc. Redding & Co. carried out the work, and the high-tensile bars were supplied by the Stressteel Corporation.

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Multiple-story Garages.

On this and the next three pages are illustrated some modern multiple-story garages. These, with many other photographs of garages and concrete roads in various parts of the world, were shown at a comprehensive exhibition of photographs, models, and drawings, arranged by the Cement and Concrete Association and held in London last month, showing how the traffic problem is being dealt with in many countries.

Fig. 1.—Garage in Chicago, U.S.A.

Fig. 2.—Garage with Hoist Tower, Chicago, U.S.A.

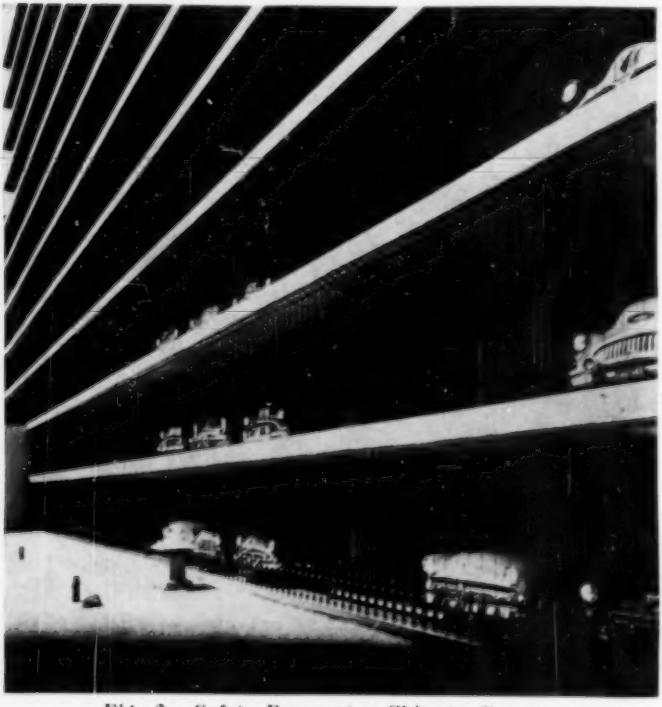

Fig. 3.—Safety Fence at a Chicago Garage.

Fig. 4.—Garage at Salt Lake City, U.S.A.

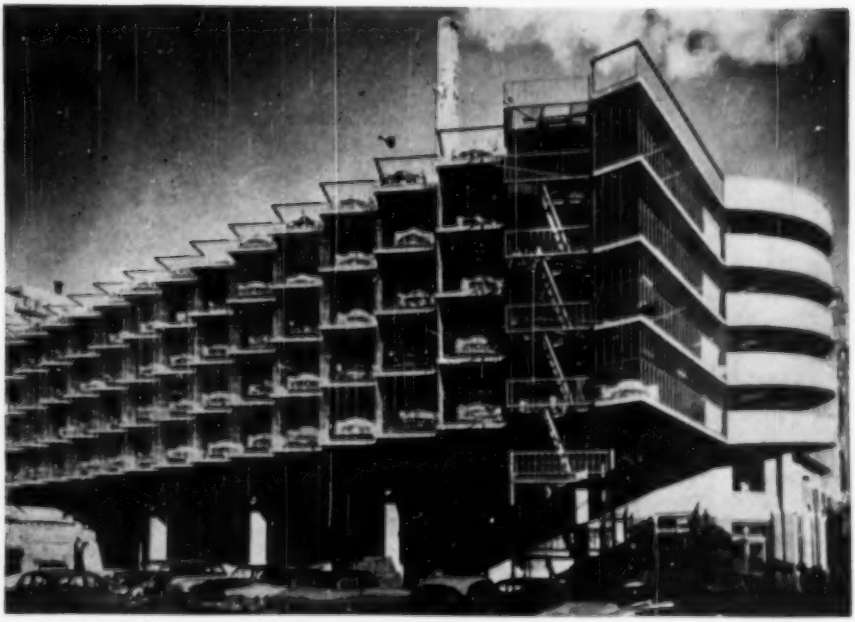

Fig. 5.—Garage at Salt Lake City, U.S.A.

Fig. 6.—Garage at Tulsa, U.S.A.

Fig. 7.—Underground Garage, Chicago, U.S.A.

Design of Tee-Beams by a Semi-Graphical Method.

By YÜKSEL SEZGINER, Dipl. Ing. (Turkey).

THE graphical method of designing singly-reinforced rectangular beams described by Mr. J. S. Savona in this journal for May, 1954, can also be applied to tee-beams, as in the following. The symbols used are :

M,	applied bending moment.
$n = n_1 d$,	depth of neutral axis.
A_T,	area of tensile reinforcement.
b,	breadth of the rib.
b_e,	effective breadth of the flange.
d,	effective depth of the beam.
d_s,	thickness of slab.
$\phi = \frac{d_s}{d}$,	the ratio of the depth of the slab to the effective depth of the beam.
$P_T = \frac{A_T}{b_e d}$,	proportion of tensile reinforcement.
c,	maximum compressive stress in the concrete.
t,	maximum tensile stress in the steel.
$m = \frac{E_s}{E_c}$,	modular ratio.
$r = \frac{t}{c}$,	ratio of maximum stresses.

Neglecting the stresses in the rib, the equation of equilibrium between the applied bending moment and the internal moment (*Fig.* 1) is

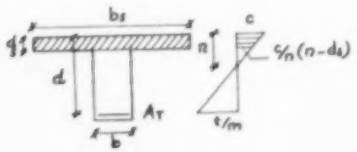

Fig. 1.

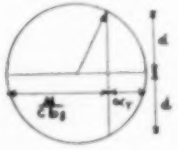

Fig. 2.

$$M = b_e\left[\frac{c}{n}(n - d_s) + \frac{c}{2}\right]d_s\left[d - \left\{\frac{d_s}{3}\left(\frac{3cn - 2cd_s}{2cn - cd_s}\right)\right\}\right]$$

$$= cb_e d^2\left[\left\{\frac{2n_1 - \frac{d_s}{d}}{2n_1}\right\}\frac{d_s}{d}\left\{1 - \frac{d_s}{3d}\left(\frac{3n_1 - 2\frac{d_s}{d}}{2n_1 - \frac{d_s}{d}}\right)\right\}\right]$$

$$= cb_e\frac{d^2}{\alpha_T} \text{ if } \alpha_T = \frac{1}{(2n_1 - \phi)\phi\left\{1 - \frac{\phi}{3}\left(\frac{3n_1 - 2\phi}{2n_1 - \phi}\right)\right\}}$$

Therefore $$\frac{M}{cb_s} = \frac{d^2}{\alpha_T}.$$

This equation can be shown diagrammatically as in *Fig.* 2.

Values of α_T may be related to n_1, which, for problems where the maximum stresses only are given, is obtained from $n_1 = \frac{m}{m + r}$, and, where the dimensions of the beams and areas of reinforcement are given, is obtained from $n_1 = \frac{mP_T + \frac{\phi^2}{2}}{\phi + mP_T}$. Therefore graphs may be drawn giving values of α_T in relation to ϕ and n_1 (*Fig.* 3), and ϕ and P_T (*Fig.* 4).

The same method may be applied to tee-beams taking account of the stresses

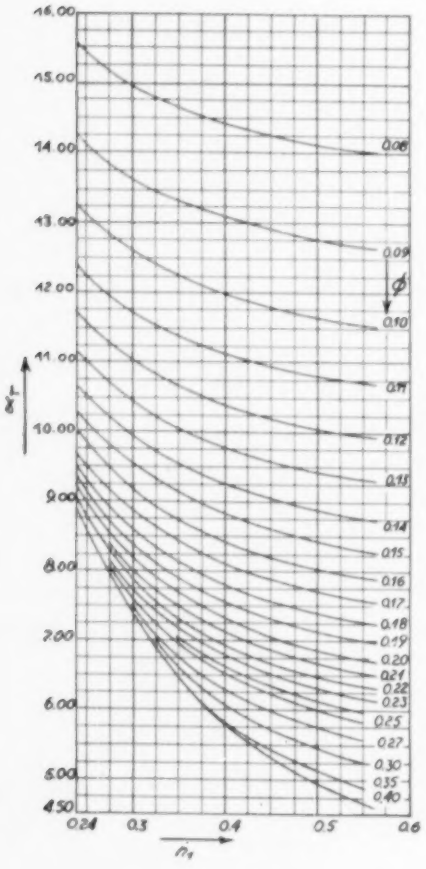

Fig. 3.

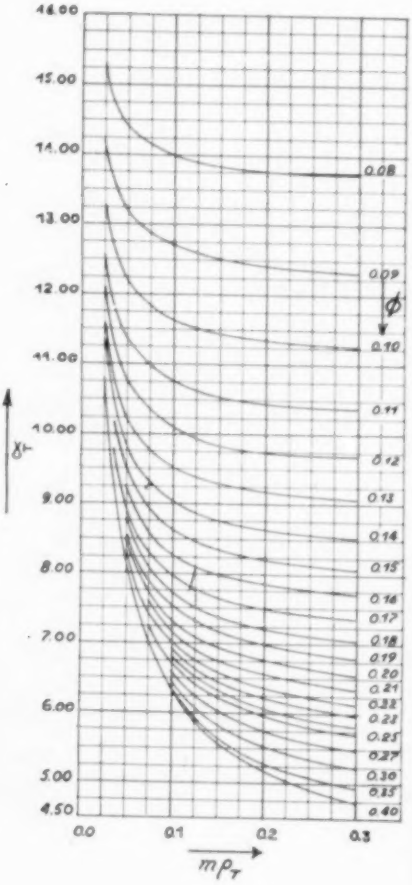

Fig. 4.

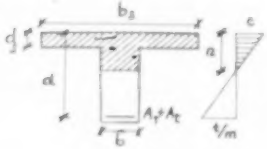

Fig. 5.

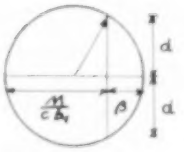

Fig. 6.

in the rib (*Fig.* 5). The applied bending moment is expressed as $M = M_1 + M_0$, where $M_1 = cb_1\frac{d^2}{\alpha_T}$ and $M_0 = cb\frac{d^2}{\alpha}$, in which $b_1 = (b_s - b)$ and α is a constant for a singly-reinforced rectangular beam (see Mr. Savona's article).

Thus $$M = cd^2\left(\frac{b_1}{\alpha_T} + \frac{b}{\alpha}\right) = cd^2b_1\left(\frac{1}{\alpha_T} + \frac{b}{b_1}\frac{1}{\alpha}\right).$$

Let $\frac{1}{\beta} = \frac{1}{\alpha_T} + \frac{b}{b_1}\frac{1}{\alpha}$, then $\frac{M}{cb_1} = \frac{d^2}{\beta}$ in which β is a constant similar to α and α_T. The diagram expressing the equation is shown in *Fig.* 6.

Examples.

EXAMPLE 1.—Neglecting the stresses in the rib, determine the effective depth of a tee-beam to resist a bending moment of 2,700,000 in.-lb. The allowable stresses are 750 lb. per square inch compression in the concrete and 18,000 lb. per square inch tension in the steel; $b_s = 60$ in.; $d_s = 4$ in.; $b = 12$ in.; $m = 15$; $r = \frac{t}{c} = \frac{18{,}000}{750} = 24$; $n_1 = \frac{15}{15 + 24} = 0{\cdot}385$.

Assume that $d = 22$ in., then $\frac{d_s}{d} = \frac{4}{22} = 0{\cdot}182$; from *Fig.* 3 $\alpha_T = 7{\cdot}84$.

$\frac{M}{b_s c} = \frac{2{,}700{,}000}{60 \times 750} = 60$. Draw the diagram in *Fig.* 7(*a*). Then, to the same scale, $d = 22$ in. as assumed. From *Fig.* 4, for $\alpha_T = 7{\cdot}84$ the value of $mP_T = 0{\cdot}087$. Thus $P_T = \frac{0{\cdot}087}{15} = 0{\cdot}0058$ and $A_T = 0{\cdot}0058 \times 60 \times 22 = 7{\cdot}65$ sq. in.

To solve this problem without neglecting the stresses in the rib, estimate the effective depth d as 21 in.

Then $\phi = \frac{d_s}{d} = \frac{4}{21} = 0{\cdot}19$, and, from *Fig.* 3, $\alpha_T = 7{\cdot}64$.

From the chart for rectangular beams $\alpha = 5{\cdot}96$; then

$$\frac{1}{\beta} = \left(\frac{1}{5{\cdot}96} \times \frac{12}{48}\right) + \frac{1}{7{\cdot}64} \text{ and } \beta = 5{\cdot}78. \quad \frac{M}{b_1 c} = \frac{2{,}700{,}000}{750 \times 48} = 75.$$

Draw *Fig.* 7(*b*) thus obtaining $d = 21$ in. From *Fig.* 4, with $\alpha_T = 7{\cdot}64$ and $mP_T = 0{\cdot}089$, obtain $A_T = \frac{0{\cdot}089}{15} \times 48 \times 21 = 5{\cdot}98$ sq. in. For $\alpha = 5{\cdot}96$ and

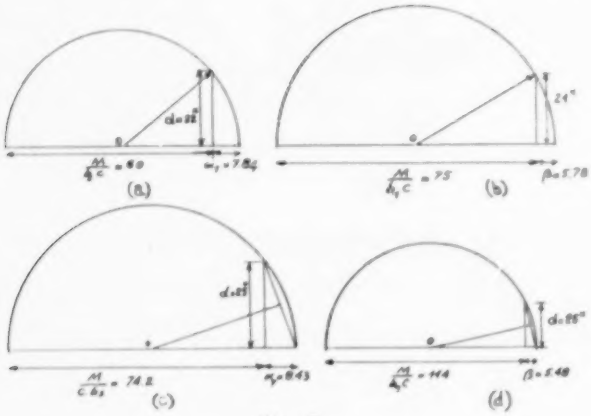

Fig. 7.

$mp = 0{\cdot}12$, obtain from the chart for rectangular beams $A_t = \frac{0{\cdot}12}{15} \times 12 \times 21 = 2{\cdot}02$ sq. in. Therefore the required total area of steel is $5{\cdot}98 + 2{\cdot}02 = 8$ sq. in.

EXAMPLE 2.—Calculate the stresses in the concrete and the steel, neglecting the stresses in the rib, if $M = 1{,}900{,}000$ in.-lb; $b_s = 35$ in.; $d_s = 4$ in.; $b = 10$ in.; $d = 25$ in.; area of tensile steel $= 5$ sq. in.; $m = 15$.

$$P_T = \frac{5}{35 \times 25} = 0{\cdot}00572; \quad mP_T = 0{\cdot}0857;$$

from *Fig.* 4 $\alpha_T = 8{\cdot}43$, since $\frac{d_s}{d} = \frac{4}{25} = 0{\cdot}16$.

Thus by drawing a diagram such as *Fig.* 7(*c*), $\frac{M}{cb_s} = 74{\cdot}2$ is obtained, whence

$$c = \frac{1{,}900{,}000}{35 \times 74{\cdot}2} = 732 \text{ lb. per square inch.}$$

For $\alpha_T = 8{\cdot}43$, *Fig.* 3 gives $n_1 = 0{\cdot}40$; as $t = mc\frac{1 - n_1}{n_1}$, then

$$t = 15 \times 732 \times \frac{1 - 0{\cdot}4}{0{\cdot}4} = 16{,}460 \text{ lb. per square inch.}$$

If the stresses in the rib are not neglected this example may be solved as follows. The tensile steel is considered in two parts, one being the steel in a tee-beam with an effective flange width of $b_s - b = 35 - 10 = 25$ in., and an effective depth of 25 in., the other being the steel in a "rectangular" beam 10 in. wide by 25 in. deep. Estimating the steel in the "rectangular" beam as $A_t = 1{\cdot}87$ sq. in., $p = \frac{1{\cdot}87}{25 \times 10} = 0{\cdot}00748$; $pm = 0{\cdot}1121$; $\alpha = 6{\cdot}11$; and $n_1 = 0{\cdot}374$.

Then, if $A_T = 3{\cdot}13$ sq. in., $P_T = \frac{3{\cdot}13}{25 \times 25} = 0{\cdot}00501$ and $mP_T = 0{\cdot}0751$. With $\phi = 0{\cdot}16$ and $\alpha_T = 8{\cdot}57$ the same value is obtained for $n_1 = 0{\cdot}374$. Then

$\frac{1}{\beta} = \frac{1}{8{\cdot}57} + \left(0{\cdot}4 \times \frac{1}{6{\cdot}11}\right)$ and $\beta = 5{\cdot}48$. By drawing *Fig.* 7(*d*), $\frac{M}{b_1 c} = 114$ is obtained, when $c = \frac{1{,}900{,}000}{25 \times 114} = 667$ lb. per square inch and

$$f = 15 \times 667 \left(\frac{1 - 0{\cdot}374}{0{\cdot}374}\right) = 16{,}770 \text{ lb. per square inch.}$$

Making Trial Mixtures of Concrete by Adding Aggregates to Grout.

THE method described in the following was used by Messrs. W. S. Atkins & Partners, consulting engineers, for proportioning the concrete in the construction of the Abbey Steel Works in South Wales.

As in other methods, the water-cement ratio to produce the required compressive strength is first decided. The water absorbed by the aggregates is allowed for, either by preparing them in a saturated surface-dry state or, more easily and accurately, by drying the aggregates and adding the water to be absorbed by them to the water necessary to give the required strength and workability. The trial batch of concrete must be sufficiently large compared with the size of the coarse aggregate, otherwise the results will be influenced by the conditions at the surface of the batch where a higher proportion of fine aggregate is required. An empirical rule is that the volume of the trial mixture be at least the cube of eight times the diameter of the largest stones. When concrete is to be placed in confined spaces, however, the surface effect requires that the size of the sample should be limited. For example, 2 cu. ft. would suffice if the concrete were required for a column 12 in. square.

A quantity of cement, estimated to be sufficient for a trial mixture of the required size, is weighed. The calculated amount of water required is also weighed, and generous amounts of sand and stone are weighed and kept in separate containers.

The cement and water are placed in the mixer. The stone is added until the mixture contains as much as possible without being too stony. Some sand is then added to make the mixture drier and less stony. More stone is added, followed by more sand, until the mixture has the required consistency and workability. The quantities of sand and stone remaining are weighed, and the proportions of the mixture are calculated. If the mixing is done by hand, the water can be added with the first additions of sand and stone.

The method can be used for mixtures containing three types of aggregate, but it is a little more complicated and depends upon the aggregates to be used. If the three aggregates are of different sizes it is possible to proportion directly, as follows. Generous amounts of the three sizes of aggregates are weighed and put in containers. First the largest aggregate is added to the grout until it is sufficiently stony, and the medium-sized aggregate is then added until no more can be added without making the mixture too gritty. This makes it less stony, so that more stone can be added, and so on. In this way the proportions approach the stage where the mixture is sufficiently stony and gritty. Sand is then added and the process repeated until wetness, grittiness, and stoniness are satisfactory. When one is accustomed to this procedure large quantities of all the aggregates can be added at first. If two of the aggregates are not very different in size they may be added in a fixed proportion, chosen from experience or a grading curve, before starting to prepare the trial mixture. When large quantities of concrete are required, the cheapest mixture may be determined from a graph showing the cost per cubic yard of concrete plotted against the proportion of intermediate aggregate for mixtures of equal strength and workability.

[A method of preparing trial mixtures by adding sand and coarse aggregate to cement paste, as used by the Australian Department of Works, was described in our number for June 1955.]

Book Reviews.

"Analysis of Symmetric Cylindrical Shells." By J. McNamee. (London: 1955. H.M.S.O. Price 12s. 6d.)

THE author has been engaged at the Building Research Station on an investigation of ways of computing stresses in shell structures, and particularly in cylindrical shell roofs. In this book, based on lectures given at Liverpool University in the year 1953, is presented an account of a method which is related to a generalisation of the analysis of beams and slabs. The method consists of two parts, namely, first the calculation of membrane stresses satisfying internal equilibrium but which, in general, leave unbalanced forces and moments at the edges, and secondly the calculation of the values of these forces and moments and their effect on the structure. The second part entails the solution of a set of simultaneous linear differential equations, and for this purpose the author uses matrix algebra which, he states, has considerable advantages compared with other methods in enabling the arithmetic to be systematically checked. A numerical example illustrates the application of the method.

"Precast Concrete." By K. Billig. (London: Macmillan & Co., Ltd. 1955. Price 32s.)

IT is only in recent years, in this country at least, that precast concrete has become an important method of construction other than for floors and roofs. This development has been due in part to the shortage of timber in recent years, but largely to the combination of precast concrete with prestressing. The disadvantages of precast concrete are largely the difficulties of joining the component members and of handling heavy members. Prestressing enables the members to be joined simply and neatly and, while the entire member can usually be of a smaller cross section in prestressed concrete than in reinforced concrete, the ease with which they may be joined enables members of any size to be made of relatively small and easily-handled parts. It is opportune therefore to have a book dealing with precast structural concrete for, with the exception of a book in the German language, there is little information available other than that scattered in technical journals.

The author treats the subject in four main divisions, namely, design and production; component members; joints; and examples of precast building and civil engineering structures.

In the chapters on design the differences between precast and cast-in-situ work are emphasised and the author has wisely avoided writing elementary chapters on the computation of stresses and dimensions. The part dealing with joints gives many examples; some are useful, but some may well be described as fantastic in their complexity. Examples of components and structures are drawn from many countries and give a broad view of the possibilities of using precast concrete, while avoiding the more publicised structures with which the reader may be assumed to be acquainted. The advantages of combining precast with in-situ concrete are discussed and illustrated. The book is interesting as a first attempt to deal exhaustively with the subject in the English language; it will be useful to students wishing to specialize in this method of construction but, by reason of its generality, it is of doubtful value for reference in the office. —J. E. G.

"Das Knicken gerader Stäbe mit linear veränderlicher Längskraft im elastischen und unelastischen Bereich," by Fritz Reuttzhuber. **"Tafeln fur gleichmässig Vollbelastete Rechteckplatten,"** by Fritz Czerny. Bautechnik-Archiv, issue II. (Berlin: Wilhelm Ernst & Sohn. 1955. Price 11.80 D.M.)

THE first part of this book deals with the theory of buckling of struts in structural steelwork and in aircraft construction. The second part gives tables and diagrams for the design of rectangular slabs carrying a uniformly-distributed load over the entire surface and considers various combinations of free and clamped edges. The method is strictly mathematical, being based upon the various values of Poisson's ratio for concrete, cast iron, and steel. While this factor has some influence on the span-moments it does not affect the edge-moments. The usual assumption that Poisson's ratio is zero for reinforced concrete, although approximate, gives results sufficiently accurate for practical purposes. The value of this paper lies in its appreciation of the fundamental principles of the problem.

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Strengthening a Reinforced Concrete Slab.

DURING the construction of a four-story reinforced concrete framed building in Egypt serious cracking occurred in the slabs adjacent to the beams. This was found to be due to lack of reinforcement in the tops of the slabs over the beams and insufficient reinforcement in the bottoms of the slabs at midspan to resist the bending moments on the slabs.

Before deciding upon a method of strengthening the slabs, tests were made to measure their elastic and plastic deflections in the cracked condition. To measure the deflection, holes were made through the centres of the slabs and ¾-in. diameter tubes passed through and fixed to the slabs. A ⅜-in. diameter rod was passed through the tube and its lower end supported on a timber frame carried on the floor below. A dial-gauge was supported on the slab, the plunger of the gauge bearing on the upper end of the ⅜-in. rod; consequently any movement of the slab was recorded on the gauge. The elastic and plastic deformation immediately after loading was 0·4 in., and increased to 0·5 in. after 24 hours. The residual deflection was 0·185 in.

The method of strengthening consisted in casting a 2-in. reinforced slab about 7 ft. wide over the beams. To anchor the reinforcement to the existing slab, holes, 8 in. by 2 ft. on plan, were cut at intervals of 2 ft. 4 in. on each side of the beams (*Fig.* 3). The reinforcement consists of groups of two ⅜-in. diameter bars at 2-ft. 4-in. centres with their ends bent down into the holes through the existing slab, smaller bars being provided between these bars as shown in *Fig.* 1. Before casting the 2-in. slabs the floors were propped to prevent deflection due to their

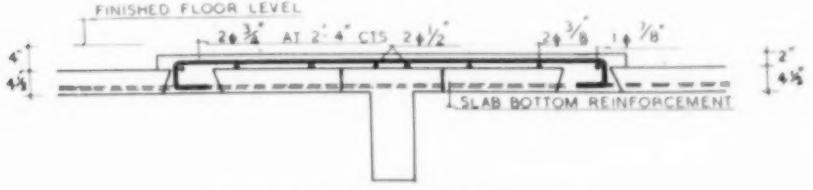

Fig. 1.—The Additional Reinforcement.

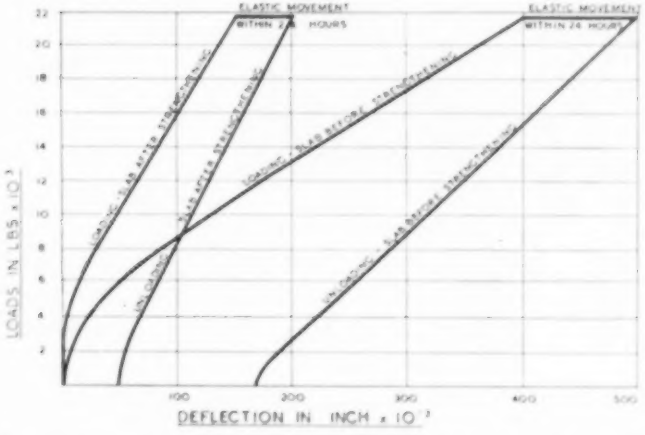
ELASTIC MOVEMENT WITHIN 24 HOURS
ELASTIC MOVEMENT WITHIN 24 HOURS
LOADING SLAB AFTER STRENGTHENING
LOADING SLAB BEFORE STRENGTHENING
LOADING SLAB AFTER STRENGTHENING
LOADING SLAB BEFORE STRENGTHENING
LOADS IN LBS x 10³
DEFLECTION IN INCH x 10⁻³

Fig. 2.—Comparison of the Deflections Before and After Strengthening the Slab.

own weight; the props were left in position for seven days after concreting. Further tests were made after the strengthening had been carried out. The same load was used as in the first tests, and a comparison of the deflections before and after strengthening is shown in *Fig.* 2, from which it is seen that the elastic and plastic deformations have been considerably reduced. The addition of the 2-in. slab does not interfere with the levels of the finished floor which was previously covered with wood or cement tiles.

The investigation and the strengthening of the floor were carried out under the direction of Professor H. F. El-Baroudy,

Fig. 3.

Ph.D., A.M.I.C.E., Faculty of Engineering, Alexandria University, Egypt.

A Large Prestressed Roof in the U.S.A.

WHAT is claimed to be the largest prestressed concrete roof in America has been constructed for Boeing Airplane Co. at Seattle. The building is 600 ft. by 484 ft. on plan, and the flat roof is designed to be used as a car park.

The roof consists of prestressed precast beams supporting a cast-in-place reinforced concrete slab. The main beams have a span of 44 ft. and the secondary beams span 40 ft. at 14-ft. 8-in. centres, giving a column grid of 44 ft. by 40 ft. and continuous spans of 14 ft. 8 in. for the slab. The Lee-McCall system was used for all the beams, the main beams being prestressed with $1\frac{1}{8}$-in. diameter bars and the secondary beams with three $1\frac{1}{8}$-in. bars. The 600 beams were made and erected in 75 days.

The main beams were made continuous for live load by mild steel reinforcement over the supports in the thickness of the roof slab. The method of providing a bearing on the main beams for the

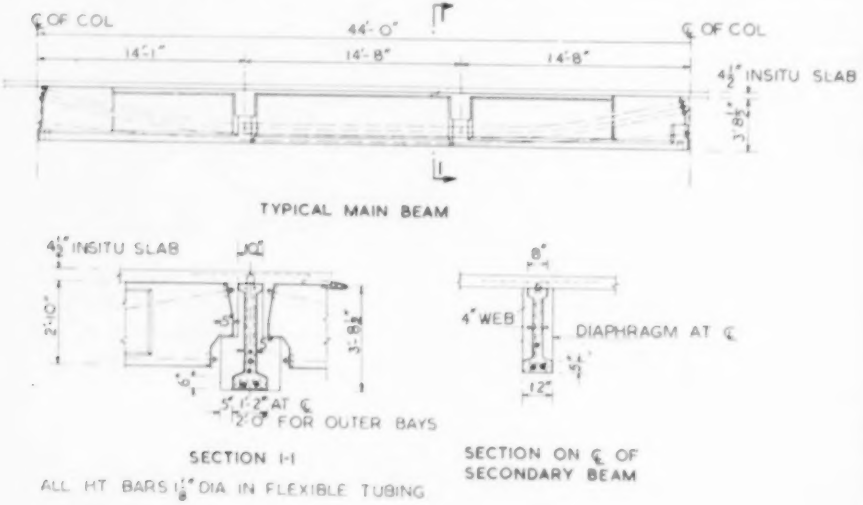

Fig. 1.

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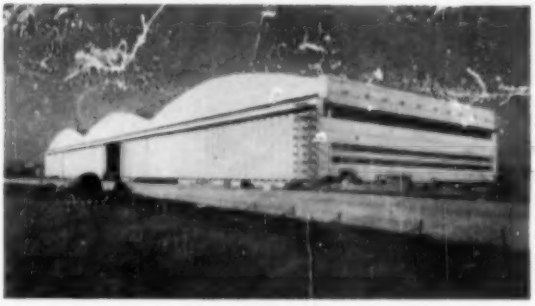

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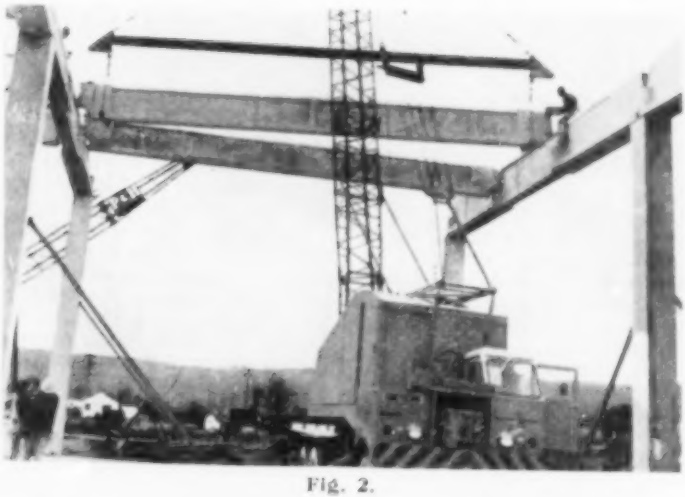
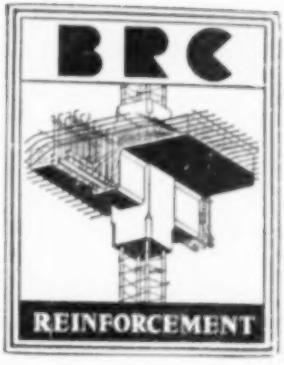


Fig. 2.

Fig. 3.

secondary beams is shown in *Figs.* 1 and 2. All the ducts for the bars were formed by flexible metal tubing in which the bars were inserted before the placing of the concrete commenced, and the bars were grouted after they were tensioned. The concrete had a compressive strength of 5,000 lb. per square inch (tested on cylinders) after steam curing for 48 hours. A view of the roof is shown in *Fig.* 3.

The consultants are Messrs. Harmon, Pray and Dietrich, of Seattle, the general contractors Messrs. Leavitt-Chandler, and the prestressing steel was supplied by Stressteel Corporation of Wilkes-Barre, Pennsylvania (licensees in North America of McCalls Macalloy, Ltd., of Sheffield).



A Combined Reinforcement and Centering for Suspended Slabs.

An article by Mr. Bengt F. Friberg in the Journal of the American Concrete Institute for May 1954 describes a combined reinforcement and centering for concrete slabs that span in one direction.

The main reinforcement is provided by galvanised steel sheets of between 18 and 24 gauge with corrugations at about 4 in. centres and usually 1¼ in. in depth. These sheets also form the centering and are supplied in lengths to span between the supports and have an adequate bearing upon them. At present the sheets are available for spans up to 14 ft. When sheets are used side by side under a slab they are overlapped 6 in. at their sides. The sheets have a tensile strength of 80,000 lb. per square inch and the working stress is normally limited to 20,000 lb. per square inch for dead loads. The secondary reinforcement consists of cold-drawn wires placed above the sheets and welded to the crest of each corrugation. The wires are normally spaced at not less than 6 in. centres and overhang one side of the sheet for a distance equal to 30 times their diameter so as to lap with the wires on the adjacent sheet. The shearing forces are assumed to be transferred from the concrete to the sheet by these wires, which also act as would distribution steel in ordinary reinforced concrete slabs.

Fig. 1 shows the sheets in position, and concrete being placed for a 4-in. continuous slab with spans of 14 ft.; the cross-wires are at 6-in. centres in the middle of the spans and at 4½-in. and 3-in. centres near the supports.

Continuous slabs are formed by using reinforcement in the ordinary way in the top of the concrete over the supports. The effective depth of slabs is measured from the top of the concrete to the centroid of the sheets in the spans and from the centre of the reinforcement to the centroid of the sheets at the supports.

Tests have been made of slabs with the same effective depths, one reinforced with bars and the other with corrugated sheets having 4-gauge distribution reinforcement at 6-in. centres welded to them; the cross-sectional area of the steel (including the corrugated sheets) in each slab was about the same. The slabs spanned 8 ft. and were loaded at the third-points; the load-deflection curves are shown in *Fig.* 2. The concrete cracked at a load of 700 lb. per foot of width in the case of the slab reinforced with bars, and 2700 lb. per foot of width in the case of the slab reinforced with sheets; these loads correspond to tensile stresses in the concrete of 430 and 2000 lb. per square inch. In the slab reinforced with bars the yield-point stress

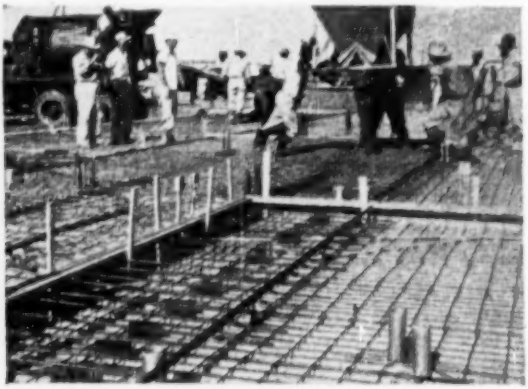


Fig. 1.—Placing Concrete on Combined Reinforcement and Centering.

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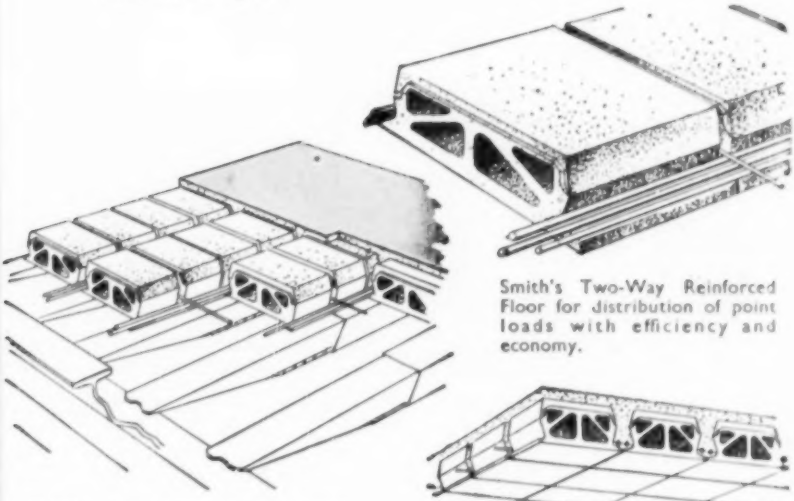

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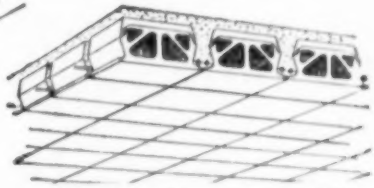

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was 50,000 lb. per square inch, and in the slab reinforced with sheets the sheet was torn away from the wires when the stress in the steel was 80,000 lb. per square inch and the stress in the concrete, computed on the assumption that the stress distribution above the neutral axis was rectangular, was 3000 lb. per square inch. When the slab failed the shearing stress in the welds joining the wires to the sheets was over 2000 lb. per square inch. Tensile cracks and initial failure of the bond between the sheet and the concrete appeared to occur at the same time and it is thought that the concrete withstood the tensile stresses without cracking so long as the reinforcement remained uniformly bonded to the concrete. The cross-wires do not act in transferring shearing forces until the slab has cracked.

In continuous slabs, initial tensile cracks appeared to occur at the supports at about the same load for each type of reinforcement, but the redistribution of the bending moments after these cracks have occurred is not the same in each case. In the slabs reinforced with bars cracks occur at the supports and in the spans at about the same load, whereas in slabs reinforced with sheets the cracks at the supports occur first because the effective depth is greater in the span and the concrete in the span appears to sustain a greater tensile force before it cracks. Increases in load increase the bending moment on the span, but the bending moments at the supports do not increase in the same proportion, the cracks over the supports acting as plastic hinges. This was confirmed when a continuous slab of two spans was loaded at the third-points in one span and the load at the free end of the span was measured; this load was one-third of the load at the third-points until the slab cracked over the support and thereafter was about 45 per cent.

The ratios of spans to effective depths and the coefficients for positive and negative bending moments suggested for slabs constructed with this type of reinforcement are similar to those used in normal British practice. Practical considerations indicate that the depth of slabs should not be less than 3½ in.

As the design of slabs reinforced with sheets is based upon the transfer of the shearing forces to the sheets it is necessary to ensure that there are enough welds of sufficient strength to effect the transfer. In some cases it is necessary to reduce the spacing of the wires to achieve this.

Temporary supports to the sheets while the concrete is hardening should normally not be more than 5 ft. to 6 ft. apart, depending upon the thickness of the slab and the allowable deflection of the sheets. The writer recommends that attention should be given to the spacing of temporary supports in long spans, as large

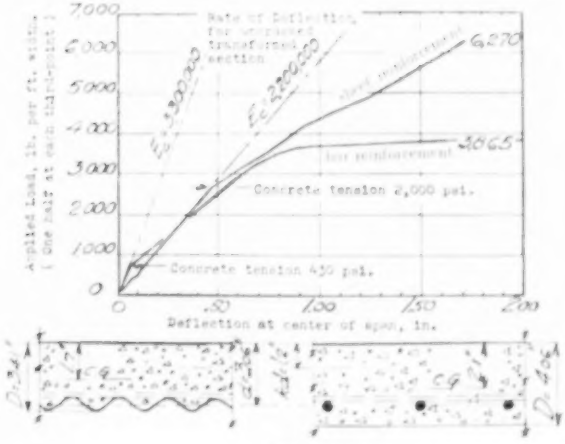
Rate of Deflection for uncracked transformed section
Steel reinforcement
6270
3065
bar reinforcement
Concrete tension 2,000 psi.
Concrete tension 450 psi.
Applied load, lb. per ft. width (one half at each third-point)
Deflection at center of span, in.

Fig. 2.

variations in the deflection of the sheets may occur with relatively small changes in the spacing of the supports.

Holes in slabs are made by forming their sides with sleeves tied to the cross-wires and cutting the sheet after the concrete has hardened. Where this type of reinforcement is used in conjunction with reinforced concrete beams the ends of the sheets project between 1 in. and 1½ in. into the sides of the beams so that they do not interfere with the reinforcement. When concrete slabs are supported by steel beams the sheets are welded to the steel-work.

Fire-resistance tests made on slabs constructed with corrugated sheets are said to have proved satisfactory. Increased fire resistance may be obtained by a light-weight plaster on laths fixed to the under-side of the corrugated sheets or by a suspended ceiling.

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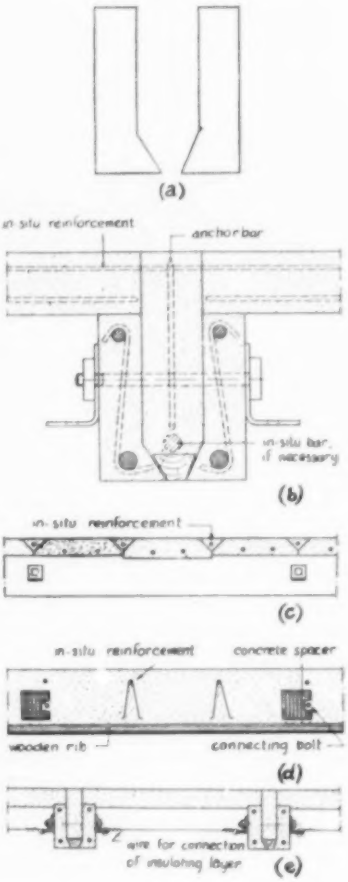
In a new system of reinforced concrete floor construction developed in Germany, reinforced concrete beams shaped as at (*a*) in *Fig.* 1 are erected in pairs and closed at the bottom by a rib of wood or concrete as shown at (*b*). Each beam is reinforced with two bars of the required diameter and these are connected by hooks also shown at (*b*). The distance between these pairs of precast members can be varied so that when the space between them is filled with concrete a composite beam of any desired width is formed. The precast beams are kept the required distance apart by concrete spacing blocks held in position by bolts passing through holes in the beams and the spacers.

When the beams are in position precast floor slabs are placed to span between them (*c*) and (*d*). A bar is then placed in the vee-grooves formed between the slabs, anchor-bars are suspended from this reinforcement into the spaces between each pair of beams, and reinforcement placed between the pairs of beams if this is required.

The spaces between the pairs of beams and the grooves between the floor slabs are then filled with concrete, and a topping may be laid over the whole of the floor.

The wooden rib in the bottom of the beam is for the purpose of fixing a suspended ceiling. If desired, ell-shaped steel angles can be used with the bolts positioning the spacing blocks to carry wires to support an extra layer of insulation (*e*). The foregoing is from "Polytechnisch Tijdschrift" for March 31, 1955.



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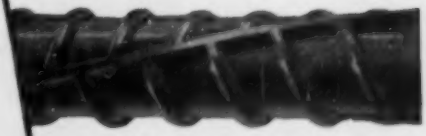

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